

# **ANALISI SPERIMENTALE DEL COMPORTAMENTO AL FUOCO DI COLONNE CIRCOLARI IN ACCIAIO AD ALTA RESISTENZA**

## **FIRE TESTING OF HIGH-STRENGTH STEEL CIRCULAR COLUMNS**

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### **ABSTRACT**

This paper presents an experimental-numerical study on the behaviour of High Strength Steel (HSS) columns at elevated temperature, both on Circular Hollow Sections (CHS) and on a Concrete Filled Tube (CFT). The measured yield strength of the circular sections was in the order of 820 MPa. In detail, three HSS CHS and a HSS CFT were tested under the standard ISO fire with constant eccentric compression load. The evolution of temperature and deformation patterns were measured by means of a comprehensive instrumentation made of thermocouples and displacement transducers. Numerical analyses were performed and compared with experimental data by employing stress-strain relationships of carbon steel at elevated temperatures provided by EN1993-1-2 associated with two different sets of reduction factors: i) those provided by EN1993-1-2 valid up to S460 steel grades; and ii) those proposed in the literature and based on tests on HSS.

### **SOMMARIO**

Il presente articolo presenta uno studio numerico-sperimentale del comportamento al fuoco di colonne circolari in acciaio ad alta resistenza. Le prove sperimentali sono state condotte sottoponendo alla curva di riscaldamento ISO standard tre colonne di solo acciaio e una colonna composta riempita di calcestruzzo che erano state preventivamente caricate con una forza assiale eccentrica di compressione. L'effettiva resistenza allo snervamento dell'acciaio ad alta resistenza era di 820 MPa. Durante le prove l'andamento della temperatura e degli spostamenti sono stati misurati mediante termocoppie e trasduttori di spostamento. Una serie di analisi numeriche sono state poi eseguite impiegando il legame costitutivo dell'acciaio ad alta temperatura fornito dall'Eurocodice EN1993-1-2 associato a due set di fattori di riduzione di resistenza e rigidità: a) quelli forniti dall'Eurocodice EN1993-1-2 e b) quelli ricavati in letteratura e basati su prove ad alta temperatura di acciai ad alta resistenza. I risultati numerici sono stati poi confrontati con i dati sperimentali.

# 1 INTRODUCTION

Although HSS has been primarily employed in mechanical applications, its use is becoming more common in the construction industry too, owing to its excellent structural properties. HSS is particularly advantageous for columns when the horizontal displacement of the building is not a limiting condition, e.g. in steel braced frames, or when the columns are stocky. With regard to fire resistance, circular profiles are typically employed in building typologies such as offices, car parks, large halls, etc where some degree of fire resistance is usually required. Moreover, HSSs have specific chemical compositions which primarily depend on rolling tempering techniques, element thickness and the producer. Nonetheless, EN1993-1-12 [1] related to HSS up to S700 grade does not provide any additional note on the design of steel structures subjected to fire and the designer is referred to EN1993-1-2 [2] valid up to S460 grade. Research on the behaviour of HSS under fire was carried by, for instance, Chen et al. [3] and more recently by Qiang et al. [4]. Nonetheless, the behaviour at elevated temperature of CHS and CFT columns made of HSS had not been investigated yet. The present work focuses on the experimental campaign and the related numerical analyses of HSS CHS and CFT columns subjected to fire loading.

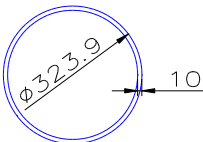
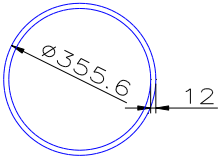
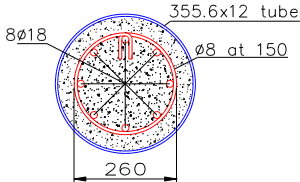
## 2 EXPERIMENTAL CAMPAIGN

### 2.1 Description of specimens

The experimental campaign consisted of: 1) three fire tests on HSS CHS which differed from one another in the cross-sectional dimensions and/or applied compression load and 2) a fire test on a HSS CFT.

The geometrical features and the actual material characteristics of the specimens are reported in Table 1. The height of around 3 meters for the columns was chosen on the basis of the dimensions of the vertical furnace. The tubes were ordered as to be made of S590 steel, which is classified as a HSS according to EN1993-1-12 [1]. However, as the tests on materials revealed, the actual grade of the structural steel was significantly higher. In the case of the CFT column, a normal weight concrete C30/37 and B450C type rebars were requested.

**Table 1:** Geometrical features and actual average material characteristics of the specimens

Name of specimen	Transverse section Diameter $D$ and thickness $t$	Length [mm]	Material
C1, C2		3150	<i>Tube</i> - $f_y = 822$ MPa, $f_u = 881$ MPa
C3		3150	<i>Tube</i> : See material properties of C1, C2
C4		3150	<i>Tube</i> : See material properties of C1, C2 <i>Rebar</i> : $f_y = 546$ MPa, $f_u = 642$ MPa <i>Concrete</i> : $f_c = 55.1$ MPa

## 2.2 Test set-up

The columns were tested in the vertical wall furnace at the fire laboratory of the University of Liege that is provided with a system capable to impose vertical loads to the specimens. With regard to the thermal boundary conditions, a closing wall was necessary in order to seal the wall furnace during the tests.

The ends of the columns were pinned in the plane perpendicular to the furnace wall, whereas in the plane parallel to the furnace wall the restraints were assumed as fixed. Moreover, an eccentricity of 10 mm was given to the load at both ends in order to force the column to buckle toward the furnace wall.

The interior side of each column was instrumented with four thermocouples Type K at three different cross-sections. In the C4 column, five more thermocouples at each section were added in order to measure the temperature of four reinforcing bars and of the concrete at the middle of the cross section. The temperature inside the furnace at different levels was monitored during each test by means of plate thermometers Type K (see Fig. 1) as well as the vertical displacement at the base and the horizontal displacement at mid-level of the column.

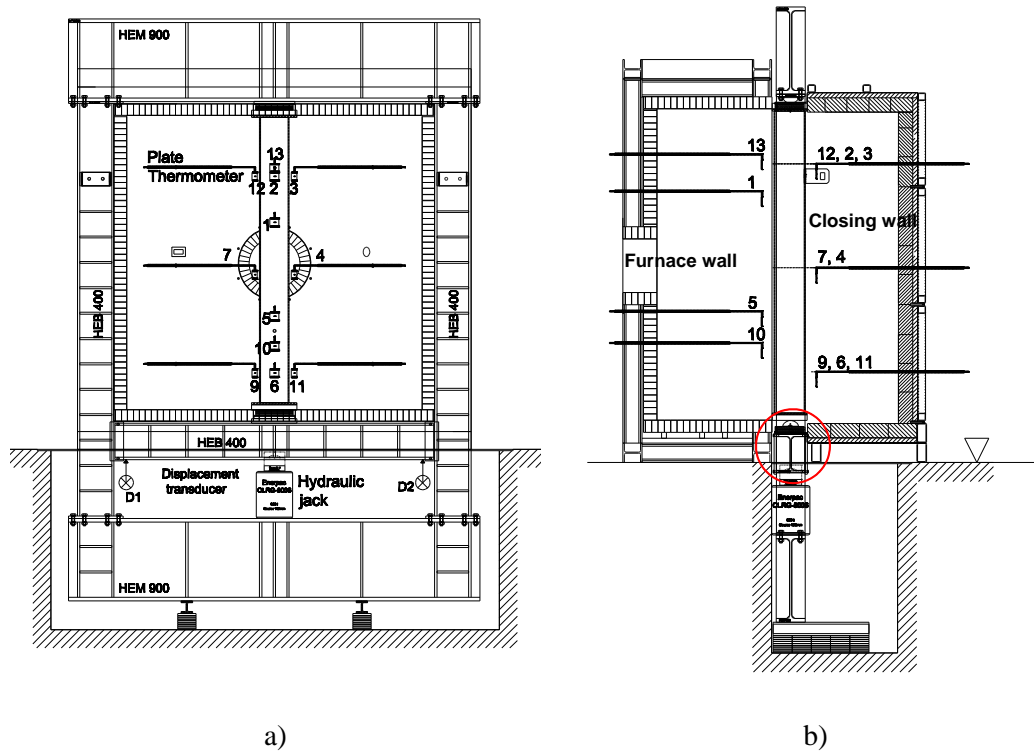


Fig. 1: Test set-up: a) front view and b) side view

## 2.3 Test procedure

The vertical loads applied during the tests and the axial capacities  $N_{b,Rd}$  of the columns, calculated by using the actual material properties and according to the relevant parts of the Eurocodes with material safety factors  $\gamma_M=1.0$ , are reported in Table 2.

**Table 2:** Applied axial load  $P$  and capacities of the columns

	C1	C2	C3	C4
Applied axial load $P$ [kN]	700	1000	1800	2000
Axial capacity $N_{b,Rd}$ [kN]	6963	6963	9286	15614
$P / N_{b,Rd}$ [%]	10.1	14.4	19.4	12.8

The load protocol followed EN1363-1 provisions which entail to maintain the load constant at least during 15 minutes, time after which the ISO 834 heating curve is applied, with the load being maintained constant until failure.

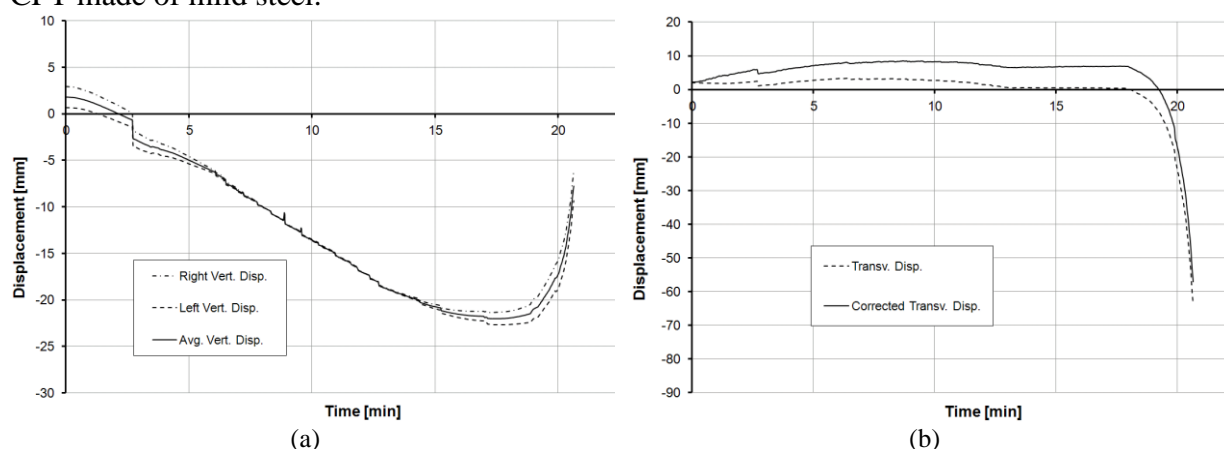
## 2.4 Experimental results

Results in terms of vertical and transverse displacement for C3 and C4 are shown in Fig. 2 and Fig. 3. The failure time was defined from  $t = 0$  defined by the EN1363-1 provisions to the instant when displacements increased with vertical asymptote. All results are extensively reported in Tondini et al. [5].

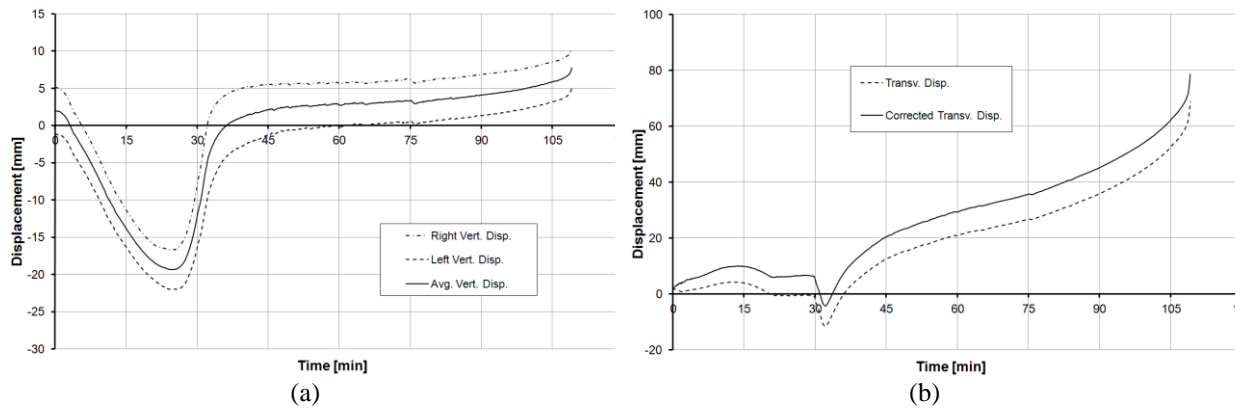
Fire resistances were 22 min, 20 min, 21 min and 108 min for C1, C2, C3 and C4, respectively.

Steel columns C1-C3 firstly exhibited shortening due to load application at room temperature; then, elongation due to thermal expansion and finally shortening at failure owing to loss of strength and stiffness at high temperature. At failure, temperatures of more than  $700^{\circ}\text{C}$  were recorded in steel columns C1-C3 that eventually buckled toward the closing wall, even though an initial eccentricity of 10 mm was given in order to force buckling toward the furnace. This behaviour can be associated with the existing temperature gradient between the furnace and the closing wall sides that was sufficiently high to move the centre of stiffness in the section toward the cooler side, thus inducing a higher eccentricity in the opposite direction of the eccentricity of 10 mm initially imposed. It was observed that failure occurred in a global buckling mode for all steel sections even though classified as class 4. In fact, no local phenomena arose in any of the columns. This may hint, as reported in [6], that slenderness limits provided for mild steel in EN1993-1-1 are too strict for very high strength steel.

The composite column C4 was provided with thermocouples attached to the interior part of the steel tube, to the steel reinforcing bars and in the concrete at the centre of the cross section. A temperature of almost  $1000^{\circ}\text{C}$  was reached on the steel tube at the end of the test. The maximum temperatures in the bars were comprised between  $300^{\circ}\text{C}$  and  $500^{\circ}\text{C}$  and at the middle of concrete of about  $160^{\circ}\text{C}$ - $180^{\circ}\text{C}$ . The evolution of the vertical displacement in a concrete filled section is completely different from that of a pure steel section: significant thermal elongation first occurs driven by the steel section, then the column shortens as the temperature in the exposed steel tube increases to very high level, leading to a deterioration of the resistance and stiffness of this tube. At this time, the load is likely to be transferred to the remaining reinforced concrete section which, being rather massive, can stand for a significant period of time. Qualitatively the behaviour observed in this test on a HSS CFT is similar to a CFT made of mild steel.



**Fig. 2:** C3 test: (a) evolution of vertical displacements and (b) evolution of the transverse displacement



**Fig. 3:** C4 test: (a) evolution of vertical displacements and (b) evolution of the transverse displacement.

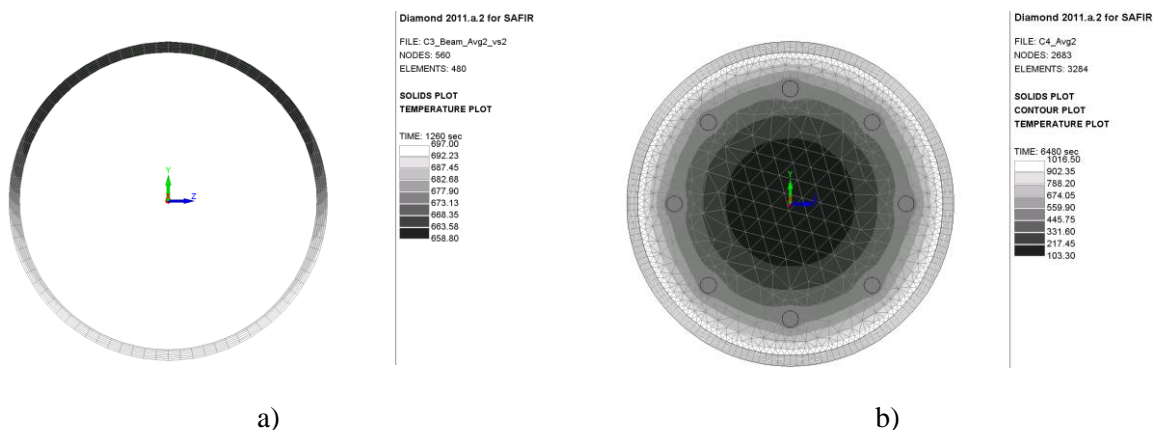
### 3 NUMERICAL INVESTIGATIONS

The experimental data, i.e. temperature distribution in the furnace and applied external load, were used as input in order to perform a series of Finite Element (FE) numerical analyses with the software SAFIR [7] with the aim to assess whether numerical simulations are capable of representing the actual behaviour observed during the tests.

#### 3.1 Thermal analyses

For thermal analyses, thermal properties for HSS, concrete and rebars were considered from the recommendations given in part 2 of the relevant Eurocodes (EN1993-1-2; EN1992-1-2; EN1994-1-2).

For the boundary conditions on the external surface of the columns, the curve describing the temperature measured in the furnace as a function of time was taken. The boundary of the section was subdivided into two parts to reflect the fact that different values of gas temperatures were recorded on both sides of the section. For the internal boundaries of C1-C3, a void condition was applied to take into account radiation and convection in the internal cavity. The results of the thermal analyses for C3 and C4 at the moment of failure are shown in Fig. 4. It is clear that the side toward the furnace (the bottom side in the Figures) is hotter than the side next to the closing wall (the top side in the Figures).



**Fig. 4:** a) C3 test: thermal analysis at 21 min; b) C4 test: thermal response at 108 min

#### 3.2 Mechanical analyses

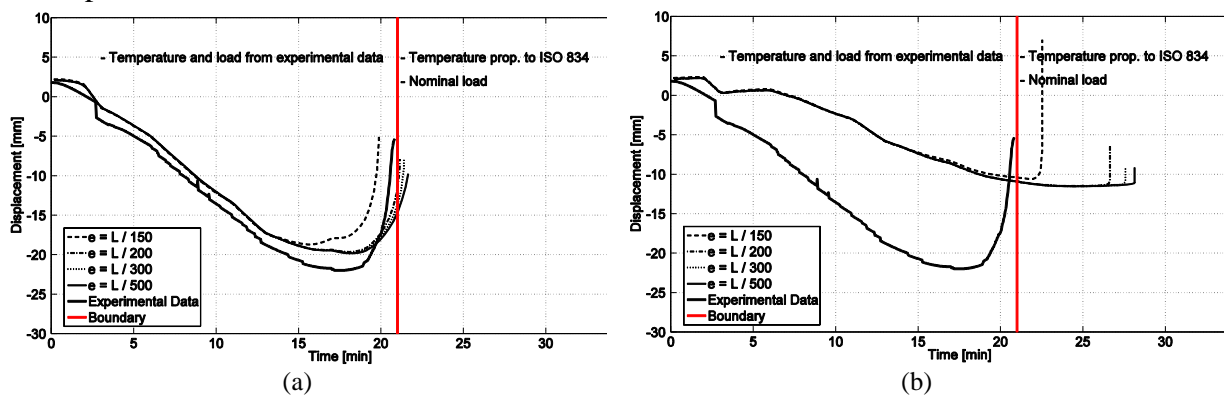
The mechanical response was investigated by performing two analyses: 1) with the stress-strain relationships of steel at high temperature and strength reduction factors from EN1993-

1-2 [2]; and 2) with stress-strain relationships of steel at high temperature from EN1993-1-2 [2] but with reduction factors proposed by Chen et al. [3], slightly adapted [5].

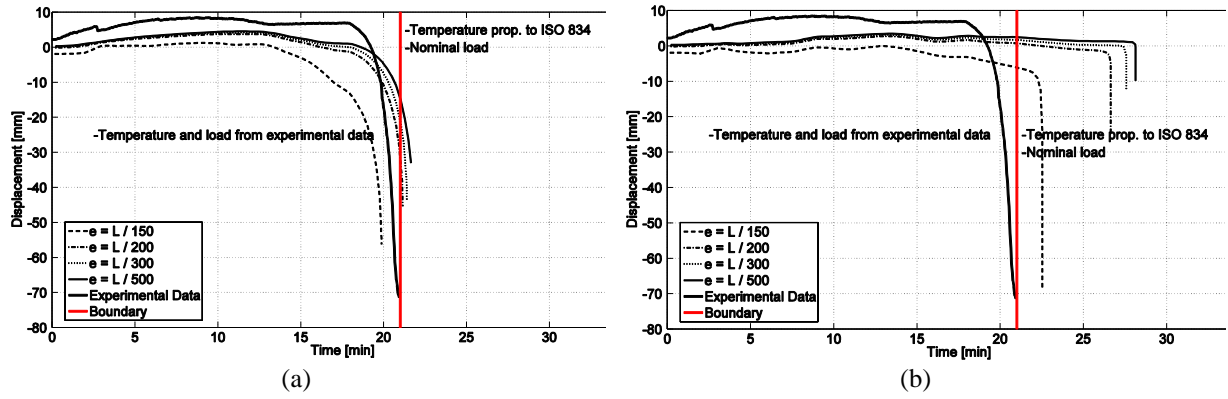
The actual axial load applied to the specimens during the tests was included in the FE model. The structural analysis was carried out by modelling the columns with 24 Bernoulli 2D beam finite elements. In the FE model, the eccentricity of 10 mm imposed in the laboratory was taken into account. Moreover, an additional initial imperfection having the shape of a half sine-wave was given. The tests of columns C1-C3 were modelled: i) with the initial imperfection toward the closing side because a direction of failure toward the closing wall was observed qualitatively in the numerical analyses only when simulating the fire tests with "negative" initial imperfection; and ii) with different values of  $e_{max}$  ( $L/500$ ,  $L/300$ ,  $L/200$  and  $L/150$ ) since it significantly influences both the direction of failure and the fire resistance time. Differently, test C4 was modelled with the initial imperfection  $e_{max} = L/500$  and directed toward the furnace wall ("positive"), toward the closing side ("negative") and with no imperfection. For C1-C3 in order to get a prediction of the fire resistance from numerical simulations, the simulations were performed until failure: a thermal analysis on each column was performed by giving as external boundary conditions: a) the experimental data of the air temperature until the time they were available and after that time b) an extrapolation of air temperatures proportional to the ISO 384 curve.

The vertical displacements are very well reproduced by the material properties of EN1993-1-2 [2] (see Fig. 5a), except for an initial setting of the test set-up during loading which is not considered in the numerical model. The properties proposed by Chen et al. [3], on the other hand, seem to underestimate the thermal elongation. For the transverse displacements compared to the experimental evidence, failure occurs later in the numerical analyses, especially when Chen et al. [3] reduction factors are used. This difference is quite significant with low initial imperfections. However, by using EN1993-1-2 [2] and with a high initial imperfection, namely  $L/150$ , the fire resistance gets closer to the experimental one. Here, the better comparisons with C3 are illustrated in Fig. 6 (refer to [5] for the complete analysis).

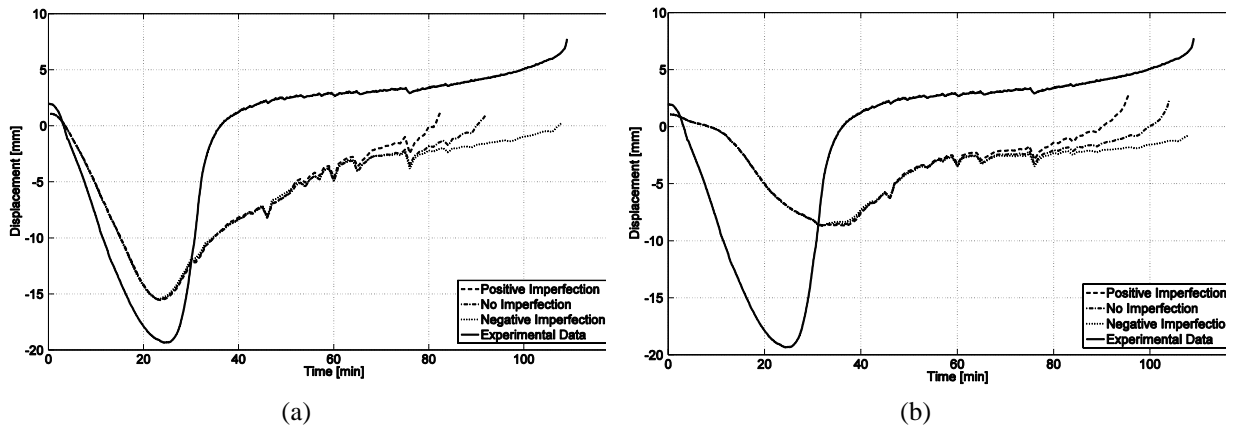
For C4 (Figs. 7 and 8) only the experimental outcomes were employed. The results of vertical displacements are also reproduced more accurately during the initial phase of the fire by the material model of EN 1993-1-2 [2], when thermal expansion of the steel tube was dominant. After the time of maximum elongation, the shortening was more pronounced in the test than in the simulations (for both set of reduction factors). In the transverse direction, the evolution of the displacement was qualitatively correct (quantitatively low) for both material models and the failure mode caused by instability toward the furnace was detected. The numerical fire resistance was comparable with that found experimentally, especially when the negative initial imperfection is considered.



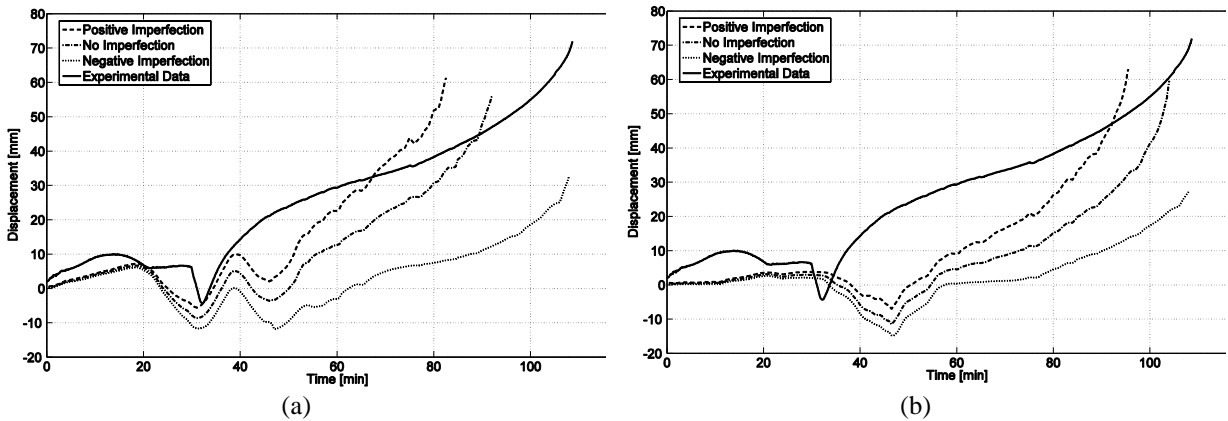
**Fig. 5.** C3: Comparison of vertical displacement between experimental data and numerical results; (a) EN1993-1-2 [2] and (b) Chen et al. [3].



**Fig. 6:** C3: Comparison of transverse displacement between experimental data and numerical results; (a) EN1993-1-2 [2] and (b) Chen et al. [3].



**Fig. 7:** C4: Comparison of vertical displacement between experimental data and numerical results; (a) EN1993-1-2 [2] and (b) Chen et al. [3].



**Fig. 8:** C4: Comparison of transverse displacement between experimental data and numerical results; (a) EN1993-1-2 [2] and (b) Chen et al. [3].

## 4 CONCLUSIONS

In this article an experimental-numerical analysis on HSS CHS and CFT columns subjected to fire loading was presented. The experimental results showed that:

- i) on the basis of the coupon tests the CHS were classified as class 4 according to EN1993-1-1. Nevertheless, no local buckling phenomena occurred in the tests. This may hint that slenderness limits provided for mild steel are too strict for VHS steels (see [6]);



ii) for HSS CHS columns, the fire resistance spanned 20-22 minutes with load ratio at ambient temperature ranging between 10.1% and 19.4%. Although the specimens were intended to buckle toward the furnace wall, because of a given initial eccentricity of 10 mm, they eventually failed toward the closing side. This behaviour was probably due to the temperature gradient in the section caused by the non-uniform heating process in the furnace that was high enough to move the centre of stiffness toward the cooler side, thus inducing a higher eccentricity in the opposite direction of the load eccentricity.

iii) for the HSS CFT column, the presence of concrete was, as expected, beneficial in terms of the fire resistance that was about 108 minutes with a load ratio at ambient temperature of 12.8 %. In this case, the eventual failure was due to global instability toward the furnace wall.

A series of numerical simulations were performed and compared with experimental evidence:

i) for the CHS columns both EN1993-1-2 [2] and Chen et al. [3] overestimated the fire resistance. Only when increasing the initial imperfection to  $L/150$  in the opposite direction of the load eccentricity the fire resistance became comparable, revealing that the initial imperfection has an important role. The vertical displacements are very well reproduced by the material properties of EN1993-1-2 [2], while the model of Chen et al. [3], on the other hand, seems to underestimate the thermal elongation.

ii) for the CFT column the numerical fire resistance was comparable with that found experimentally. In terms of displacements, in the transverse direction, the qualitative evolution was good but quantitatively low. The vertical displacements exhibited a good agreement during the initial phase of the fire by the material model of EN1993-1-2 [2].

## ACKNOWLEDGEMENTS

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## BIBLIOGRAPHY

- [1] EN1993-1-12. *Eurocode 3: Design of steel structures - Part 1-12: Additional rules for the extension of EN 1993 up to steel grades S700*, CEN, Bruxelles, 2007
- [2] EN1993-1-2. *Eurocode 3: Design of steel structures - Part 1-2: General rules - Structural fire design*, CEN, Bruxelles, 2005
- [3] Chen J., Young B. and Uy B. Behavior of High Strength Structural Steel at Elevated Temperatures, *Journal of Structural Engineering*, 132:(12), 1948-1954, 2006
- [4] Qiang X., Bijlaard F.S.K. and Kolstein H., Elevated-temperature mechanical properties of high strength structural steel S460N: Experimental study and recommendations for fire-resistance design, *Fire Safety Journal*, 55:15-21, 2013
- [5] Tondini N., Hoang V.L., Demonceau J.-F. and Franssen J.-M., Experimental and numerical investigation of high-strength steel circular columns subjected to fire loading. *Journal of Constructional Steel Research*, 80: 57-81, 2013.
- [6] Jiao H. and Zhao X.-L. Imperfection, residual stress and yield slenderness limit of very high strength (VHS) circular steel tubes, *Journal of Constructional Steel Research*, 59:233-249, 2003.
- [7] Franssen J.M.. SAFIR. A Thermal/Structural Program Modelling Structures under Fire, *Engineering Journal*, A.I.S.C., 2005;42(3), 143-158.

## KEYWORDS

High strength steel; circular hollow sections; concrete filled tubes; fire loading; experiments